INNOVATIVE USE OF PRECAST CONCRETE FOR RAILROADS AND A HEAVILY ACCELERATED DESIGN-TO-CONSTRUCTION SCHEDULE

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ABSTRACT
In August of 2006, a new rail line was deemed necessary to accommodate increased traffic on an existing route. This project was highly accelerated in that from the notice to proceed on design to end of construction was less than one year. An additional complicating aspect was a need to integrate the new bridge into the adjacent existing bridge. The project demanded a truly unique team effort amongst the owner, designer, fabricator, and contractor to meet the timeline. Critically important to the project was the availability and turnaround of the material for the superstructure. To that end, although not a traditional superstructure for railroad bridges, precast concrete I-beam construction was chosen as an unusual and unique solution for this project. The project was completed in June of 2007, a little over 10 months from notice to proceed. Presented are the challenging aspects of the project as well as how rail loading and design requirements change the traditional highway design assumptions for prestressed I-beams.

Keywords: Precast, Accelerated Bridge Construction, Rail

Background

Adger, AL is located approximately 30 miles southwest of Birmingham, AL. The rail line is on the Birmingham Mineral Subdivision that passes through Adger and is a major link between CSX Transportation and their regional connectors. Prior to the new project, a single line was carried over an unnamed creek by a two span steel girder bridge on a tangent alignment. The overall site location is shown in Figure 1. The existing steel spans consist of two riveted girders with a span length of approximately 51-ft. Girders are spaced at 9-ft. centers with an open timber deck and are supported by a combined concrete and masonry substructure.

Fig. 1, Map of the project site and aerial view of bridge

The existing abutments are traditional spread footings founded approximately 12-ft. below existing ground line and with wingwalls that are turned back at approximately 45°. The single monolithic center pier is also founded on a spread footing. The existing plan and elevation are shown in Figure 2.

In August of 2006, a new rail siding extension was deemed necessary based on an immediate need to accommodate increased traffic on the route. This project began with notice to proceed (NTP) on design in mid-August of 2006 with a target to complete construction by the end of the year. Thus, only four months were available for design, fabrication, and construction of a new bridge. However, due to unforeseen grading and rock excavation delays, the actual end of construction was completed less than one year from NTP. Project timing was key to allow the new rail traffic over the bridge and to avoid potential losses stemming from delays.

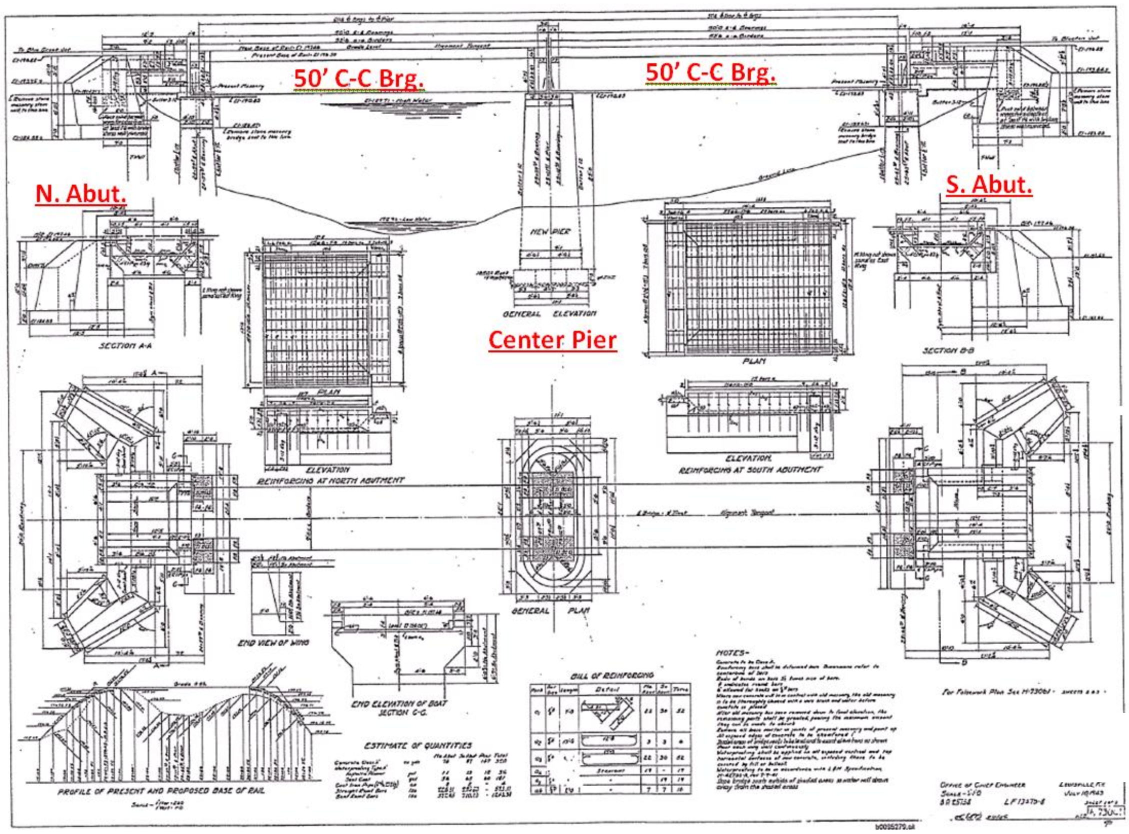

Fig. 2, Existing span

The New Bridge

Due to the schedule project coordination was essential. From the start, a few key parameters were set to accommodate the existing structure. New abutment locations were set adjacent to the existing structure and positioned so as to not substantially impact the existing foundation. Similarly, it was decided to tie into the existing center pier by extending it over a new foundation and not create a separate structure. Accordingly, the spans were set at 55'-4" based on this proposed geometry. An additional complicating factor was a need to integrate the new bridge substructure into the adjacent existing bridge.

Side by side expansions next to existing steel girders would customarily involve using steel beams or girders of similar properties for the new spans. For this project, the proposed span would have likely been a four steel beam span, as shown in Figure 3. This configuration was in fact the initial design chosen and preliminary engineered based on the project parameters and the owner's preference.

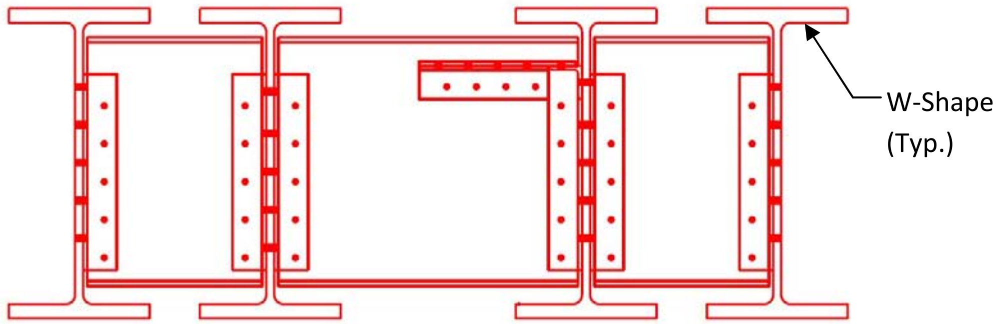

Fig. 3, Typical four beam steel span cross section

Critically important to the project was the availability of the material for a quick turnaround of the superstructure. Lead time for steel fabrication and shipping was beyond the expected three month project timeline. Therefore, either the timeline needed to be delayed or an alternate structure type was necessary. To that end, although not a traditional superstructure for railroad bridges of spans exceeding approximately 35-ft., precast concrete I-beam construction was chosen as an unusual and unique solution for the demands of this project.

Similar to new structures on existing highway alignments, the new rail structure was confined by the on-site geometry. Rails could not be raised, and the low chord was controlled by the hydrology of the existing span. This led to an initial concern for using prestressed beams: is there a beam that is both shallow enough yet strong enough to carry modern rail loads over a 55-ft. span?

Precast Design for Railroads

The precast design follows AREMA Chapter 8, Concrete Structures and Foundations, Section 17, Precast Concrete[1]. The first challenge of prestressed design for rail is the design load, a Cooper E80 Live Load (Figure 4).

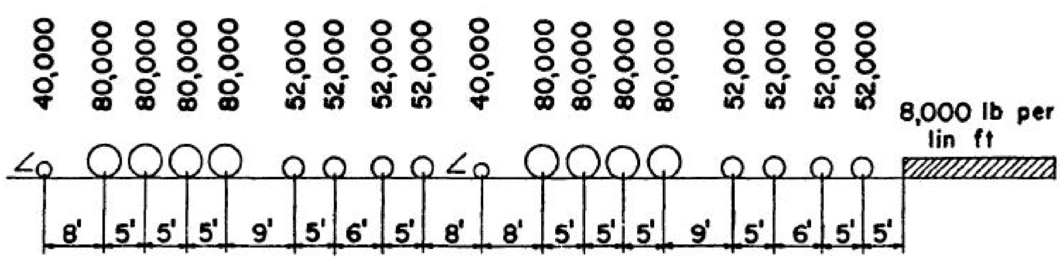

Fig. 4, AREMA Cooper E80 live load (loads above axels are in lbs.) (AREMA, 2010)

As shown in Figure 4, the Cooper E80 load introduces a significant increase over the two 32-kip axles of the AASHTO HS-25 design truck[2]. Impact is significantly increased, respectively. Most of the design parameters for rail are similar to highway requirements, such as a minimum concrete strength of 4500-psi, tendon spacing of 2-in., minimum cover requirements of 1 ½-in, and so forth. There are, however, also significant differences, as shown:

	Highway	Rail
Gamma factor (γ)	1.3	1.4
Phi factor, bending (ϕ)	1.0	0.95
Allowable Tension in the pre-compressed tensile zone	$6\sqrt{f'_c}$	0

All of these factors combined with the increases live load greatly reduce the expected span lengths for a particular precast shape.

The geometry constraints mentioned in the previous section limited the depth of the beam to no greater than 45-in. Multiple initial design runs were completed over the course of three days to verify if a prestressed section was even possible for the proposed span length. A proprietary concrete beam design program was used to analyze the structure and the load input database was modified for the Cooper E80 design load.

At an early stage in the design process, it was determined that six beams would be needed (three per rail) to handle the load and the height restrictions. Selection of a standard shape was necessary to expedite fabrication. AREMA loading assumptions also provide guidance regarding the spacing that is allowable if the beams are each to be fully utilized in the design[1]. This meant tight spacing, and AASHTO modified bulb-Tees were quickly eliminated due to the top flange width. Consequently, a 45-in standard AASHTO Type III bulb-Tee became the only option due to the height limitation and the constrained side-to-side spacing. Once this was established, the next challenge became finding and designing for a reasonable concrete strength and number of strands.

During the design process, the engineer was in frequent contact with the fabricator, Sherman Prestressed Concrete, to assess how efficient the design was for fabrication. These conversations led to agreements that, from the fabricators perspective, expedited the fabrication and allowed them to provided fabrication preferences to the designer. For example, the use of de-bonded strands and the use of higher early strength concrete to achieve the 6000-psi at release resulted from these conversations.

The AASHTO Type III beams were finalized at a 26-in. spacing, allowing a small finger-gap of 4-in. between the bottom flanges. The final concrete strength is considered high at 7000-psi, particularly when considering the standard strength used by the client is 6000-psi. This strength was necessary to appease the coupled problem of (a) requiring high pre-stressing to avoid tension in the bottom flange in service, and (b) the resulting high tensile stress in the concrete on the top flange at release. The resulting beam configuration cross section is shown adjacent to the existing structure in Figure 5.

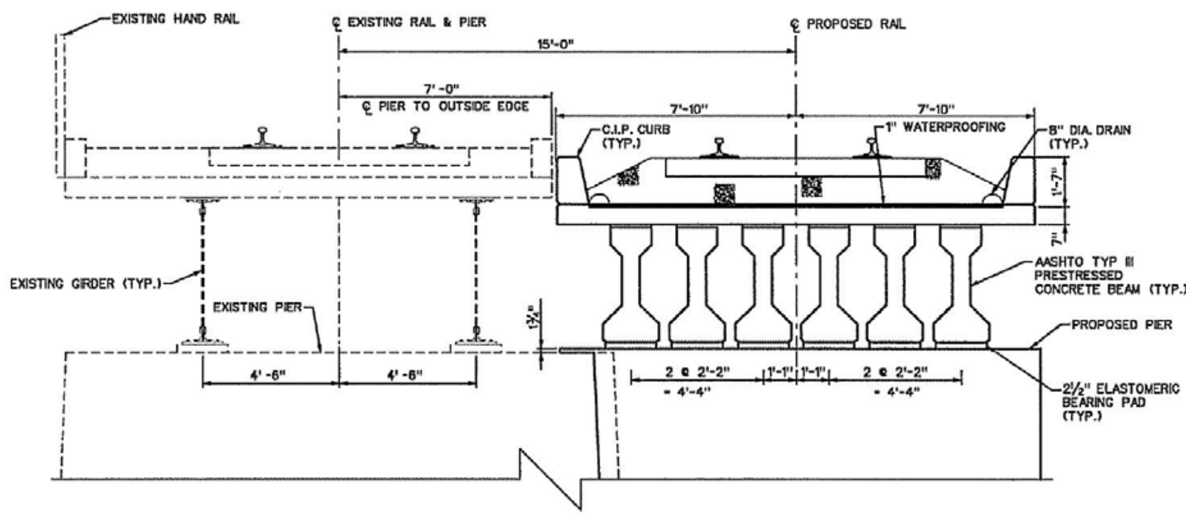

Fig. 5, Final Bridge Design Typical Section

The final design section of the Type III is shown in Figure 6. Twenty four (24) strands were required in the pattern shown. A total of four strands were deboned up to 6-ft. As soon as the designs were finalized, the fabricator immediately began production of the beams. This was a benefit specific to the communication amongst the project team.

The difference in the total live and dead loads are presented as an additional point of information. To demonstrate the difference in loading proportions, the total unfactored dead load reaction was approximately 215-kips, compared to the total unfactored live load reaction of approximately 373-kips.

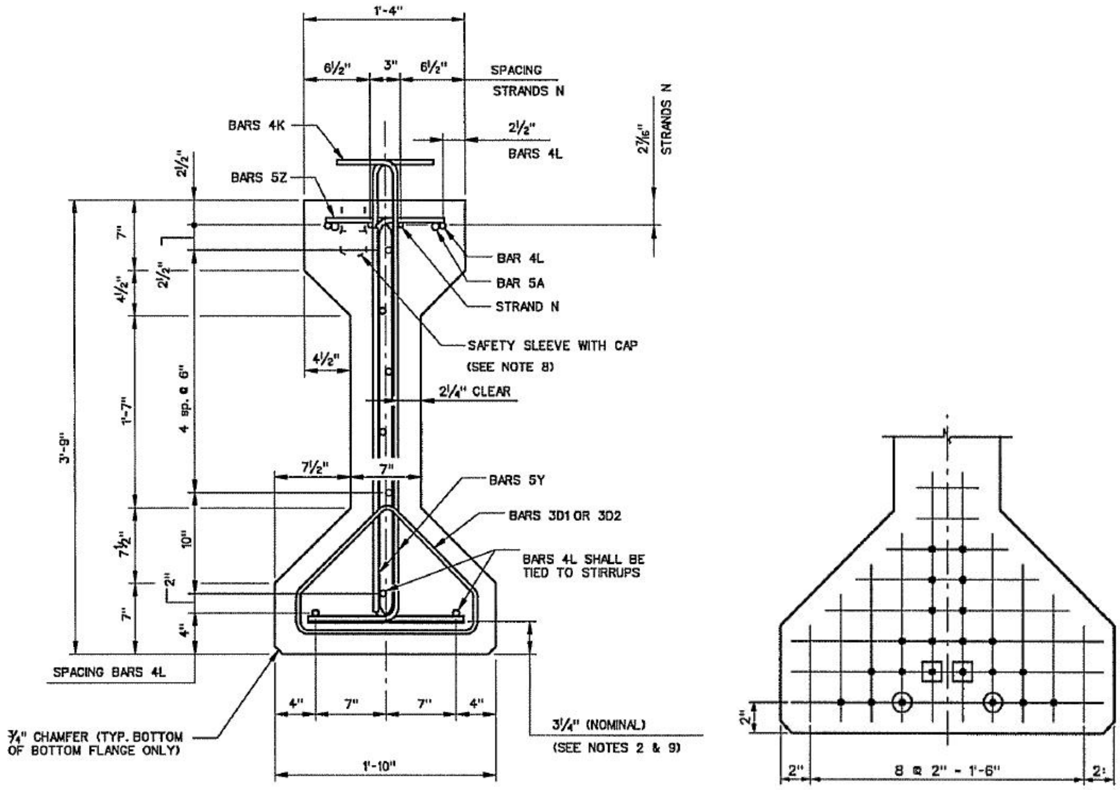

Fig. 6, Final Beam Design Section

Additional Design Considerations

The large live loads create additional challenges when siding an existing structure. Important considerations include live load surcharges at the abutments and relative deflection between the integrated substructures. Additionally, longitudinal forces at the abutments are much higher than for highway structures as a result of the longitudinal traction and breaking forces associated with the E80 design load (see Figure 7). Although much larger, these loads and deflection limits are met using traditional design techniques. Conversely, the sense of what is a *normal* design no longer applies. For example, the demands at each abutment were met with two 54-in. diameter drilled shafts and the pier was constructed by extending one 54-in. drilled shaft to the pier cap for a 16-ft. wide bridge with 50-ft. spans.

j. *Longitudinal Force.*[1]

(1) The longitudinal force for E-80 (EM 360) loading shall be taken as the larger of:

– Force due to braking, as prescribed by the following equation, acting 8 feet (2450 mm) above top of rail.

$$\text{Longitudinal braking force (kips)} = 45 + 1.2L$$

$$(\text{Longitudinal braking force (kN)} = 200 + 17.5L)$$

where L is the length in feet (meters) of the portion of the bridge under consideration

– Force due to traction, as prescribed by the following equation, acting 3 feet (900 mm) above top of rail.

$$\text{Longitudinal traction force (kips)} = 25\sqrt{L}$$

$$(\text{Longitudinal traction force (kN)} = 200\sqrt{L})$$

where L is the length in feet (meters) of the portion of the bridge under consideration

Fig. 7, Excerpt from AREMA: Longitudinal Force Equations[1]

Construction

The following is an overview of the entire timeline from start to finish. Construction began in October 2006 and important milestones included:

- Labor Notification Issued 10/12/06
- Grading Contractor Mobilized 12/20/06
- *Delay Due to Unforeseen Conditions for Track Grading*
- Bridge Work Started 3/26/07
- Track Work Started 4/27/07
- Bridge Work Complete 6/30/07
- All Work Completed 7/10/07

The timeline is represented graphically in Figure 8. Shown in the figure is the construction delay represented by the *Track Excavation*. This delay was a result of grading the right of way for the new track away from the bridge. True to schedule, however, the beams were delivered on site in time for construction December of 2010. The fabricator meeting the delivery schedule demonstrates that the bridge could have been constructed by the end of 2010, within the proposed deadline for completion.

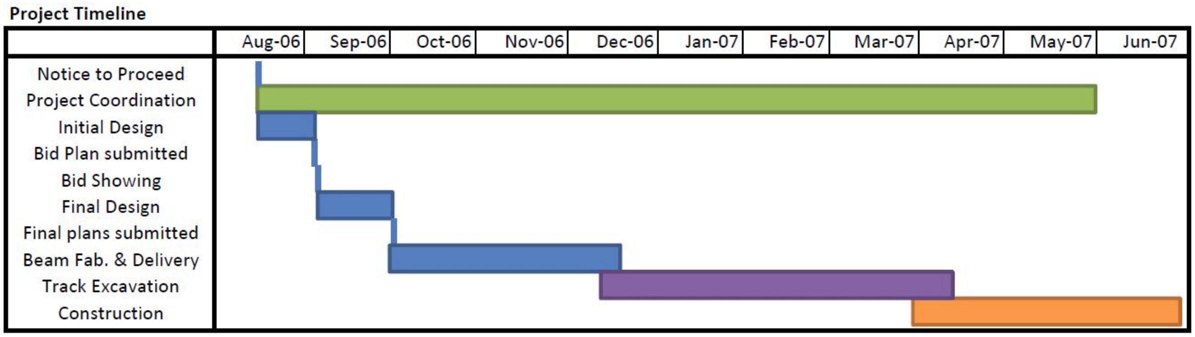

Fig. 8, Project Timeline

Even with the delay, the project was completed in nearly 10 months. Figure 9 shows the project site prior construction and Figure 10 shows the final siding bridge in place. Note the relatively tight spacing of the beams as mentioned in the design section.

Fig. 9, Project Site Prior to Construction

Fig. 10, Competed Construction

Project Team

The project team consisted of:

- Owner: CSX Transportation
- Designer: HDR Engineering
- Bridge Contractor: Scott Bridge
- Precast I-Beam Fabricator: Sherman Prestressed Concrete
- Grading Contractor: Winston Contracting
- Track Contractor: Trac-Work

The importance of Construction Coordination/Bid Meetings

On a time sensitive project with many different team members, coordinating the flow of information was critical. To accomplish this there was the traditional construction bid meeting at the site. Additionally, weekly reports were distributed to the project team during design, fabrication, and construction with contributions from each member.

The initial construction bid meeting was held at the project site with the owner and the designer to discuss the construction process. This meeting allowed the owner and designer to distribute information to the contractor prior to bidding, as well as solicit feedback from the contractors that attended. Drawings considered near-final were distributed with the understanding that the project was fluid and that the engineer would be working with team to finalize the plans. As such, the owner was able to convey the specific requirements and specifications directly to the contractors, emphasizing the schedule, and reducing the amount of confusion during construction. Environmental permitting representatives were also in attendance to assist in the decision making process and avoid any potential pitfalls. The compressed schedule was addressed and various ideas were discussed that could aide the schedule. One of the results from this meeting was the suggestion to use drilled shafts.

The weekly reports were completed by the owner with the input of the different members of the team. The reports consisted of construction and design updates, material fabrication updates, and onsite safety. It was an excellent way to not only track the project progress, but also to keep other team members aware of each others progress. Once a critical path issue was identified, having the communication already in place greatly accelerated the decision making process and sharing of the necessary information since each team member was already abreast of the current project status. Although not a design-build project, several aspects of the project were conducted in a similar manner, such as these meetings.

Superstructure Construction

Using prestressed precast concrete I-beams for this bridge allowed a few timesaving advantages for fabrication. First, engineering design time and drawing production effort was reduced by the use of standard shapes in the State of Alabama. Second, using standard shapes eliminated the need for custom formwork and allowed the fabrication to start at an earlier date. Third, standard prestressed also eliminated the need for a steel shop drawing review.

Once the design was complete, the railroad ordered the precast I-beams and the neoprene bearing pads ahead of contractor selection. This allowed the fabrication to begin while the bid process took place. As a result, the railroad delivered the precast concrete beams and bearing pads to the construction site as the contractor was expected to start work on the bridge. On their end, the contractor was only required to unloaded, handle, and place the railroad-furnished precast concrete beams and bearing pads. The contractor was required to furnish and place the CIP concrete, preformed expansion joint filler material, asphalt mastic and other remaining miscellaneous material necessary to properly set the concrete beams in place.

As stated above, even with the accelerated timeline, certain portions of the bridge needed to be cast in place. This included the drilled shaft foundations, the superstructure diaphragms, the bridge deck, and the concrete ballast retainers. The contractor was able to optimize the casting schedule by curing the CIP during the other work necessary for completion of the track portion of the project.

An added layer of complexity to rail work is that the existing rail lines commonly remain open to rail traffic during construction. Contractors are provided working “windows.” When not within a window, they must remain at a minimum clear distance from the active rail. The rail owner representative does provide active communication with the contractor by having an employee on site at all times to insure the safety of the construction site[3]. For this job, trains continued to run at regular intervals on the existing main line. During each train passage, all work adjacent to the track was stopped.

Modifying the Existing Structure

To accommodate the new bridge structure for the siding track, the existing structure for the main line track would need to be modified. First, the existing backwalls had an 8 foot wide section removed to accommodate the new ballast deck. Second, the new abutments and pier were anchored into the respective existing elements using dowel bars. An important consideration when designing the new substructure and dowels is restraining any potential differential displacement between the old and the new. This is often complicated by the fact that the original spans for railroads are frequently supported on very stiff masonry, whereas modern design generally allows for more flexing of the structure. The doweled connection at the abutments is illustrated in Figure 11.

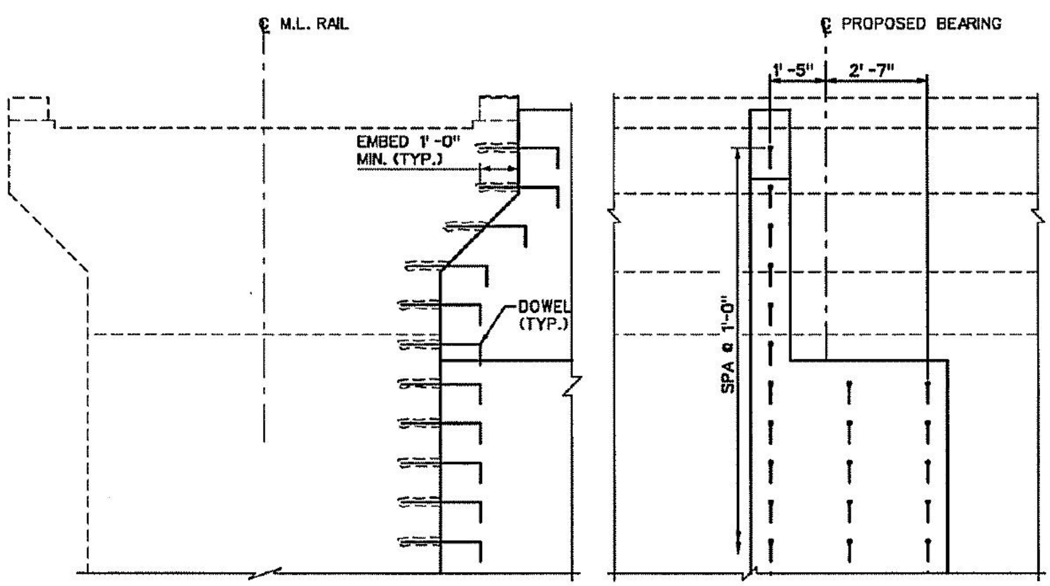

Fig. 11, Modifications to Existing Abutments

Closing Remarks

The project was completed in June of 2007, a little over 10 months from notice to proceed. Although not a design build, it demanded a team effort amongst the owner, designer, fabricator, and contractor to meet schedule. With or without a delay, the project would not have been nearly as successful without the use of precast concrete to deliver the superstructure on a critical timeline.

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